STATIC AND DYNAMIC LATERAL LOADING OF TWO PILES

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SUMMARY

Slow cyclic and dynamic load testing of two free head 450 mm diameter piles in saturated silty sands is described. Moment and deflection profiles, load/deflection behaviour and dynamic response data are presented. Representative values for the constant of horizontal subgrade reaction, n_h, are evaluated from Winkler spring and equivalent cantilever models of the piles. These are compared with predictions assessed from soil data.

INTRODUCTION

The response of pile foundations to lateral load is an important aspect of the seismic resistant design of structures and is commonly predicted using equivalent cantilever solutions or Winkler spring models. The simplifying assumption that the soil stiffness parameters are independent of the soil strains is normally used. The fundamental parameter used to model the soil behaviour is the horizontal subgrade reaction modulus, K_h. There has been very little published material to provide guidance on the choice of subgrade reaction values since the early work of Terzaghi (Ref. 1).

For design purposes, reliance is often placed upon empirical relationships to assess design subgrade reaction parameters from available investigation data such as soil bore log descriptions, penetrometer tests and sometimes pressuremeter tests. The pile testing described was performed with the objective of providing information on the reliability of such empirical relationships for the assessment of subgrade reaction parameters and the suitability of the two analysis methods for piles discussed above.

LOCATION AND GEOLOGY OF THE SITE

The site selected for the pile tests was on the flood plain of the Hutt River about 1 km upstream from where the river discharges into Wellington Harbour. The materials at the test site are in situ marine silty sands extending from 1 m to at least 8 m below the present ground level. The Wellington region has a history of tectonic uplift associated with major earthquakes, e.g. 2 m in 1855; 1.5 m circa 1400, which explains the presence of recent marine deposits in the present flood plain of the river. Overlying the marine sediments at the test site is a 0.5 m layer of fill on top of approximately 0.5 m of alluvial sediment laid down during the formation of the Hutt River delta.

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SITE INVESTIGATIONS

The results of field investigations performed at the site, which include Dutch cone penetrometer and standard penetration tests (SPT), sampling of the material from bore holes and Ménard and self-boring pressuremeter (SBP) testing are summarised in Figure 1. Only one successful test was achieved with the SBP as the equipment was not sufficiently robust to accommodate the pebble and shell fragments. Grading analysis of the materials show relatively uniform sands with varying silt content and some gravel over the depth of the piles. Typical coefficients of uniformity (D_{60}/D_{10}) for the sands were 2.5 to 6 and the D_{20} size (20% passing) varied between 0.06 mm and 0.2 mm. It was observed that these sands fall within the range of materials susceptible to pore pressure development and strain softening during dynamic loading. SPT results support this observation.

Cyclic loading tests were performed with the Ménard pressuremeter at depths of 2.35 m and 3.95 m. The cyclic moduli (tangential) were 15 x 10^3 kPa and 11 x 10^3 kPa respectively. These values are significantly higher than the first loading moduli and are similar to the 12 x 10^3 kPa first loading modulus measured with the SBP at 2.35 m. Two consolidated undrained triaxial tests with pore pressure measurement were performed at a strain rate of 0.1% per minute and these indicated soil strength parameters of $c' = 0$ kPa, $\phi' = 35°$. There was very little variation between the two series of tests which had recompacted dry densities of 1.5 and 1.65 t/m^3.

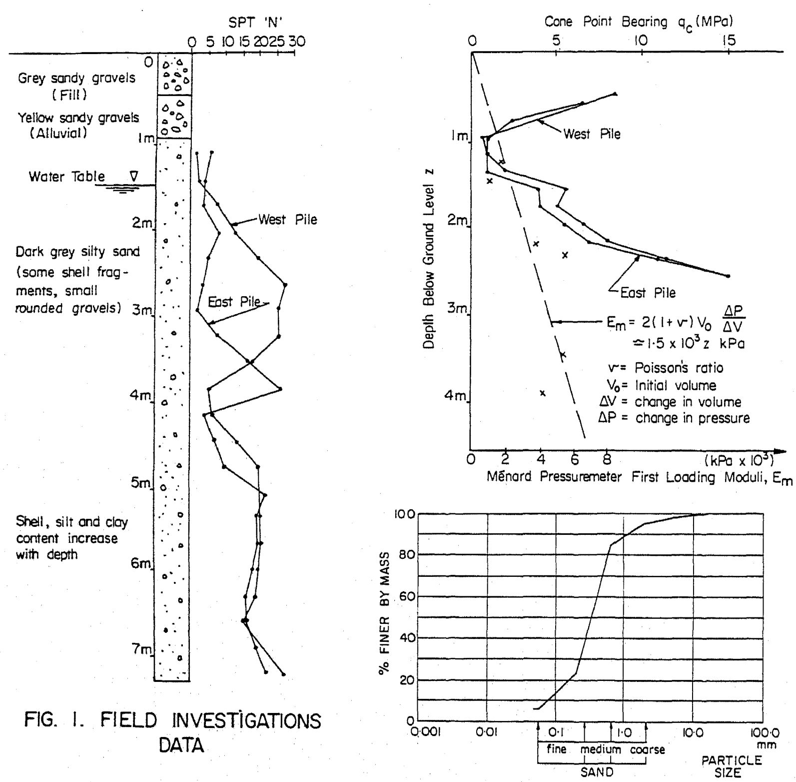

FIG. 1. FIELD INVESTIGATIONS DATA

TEST PILE ARRANGEMENT AND INSTRUMENTATION

Two 450 mm diameter steel shell piles (10 mm wall thickness) 8.25 m long were embedded 6.75 m (15D) below ground level. The 0.5 m layer of fill was removed prior to installation. Construction of the piles involved auger drilling 400 mm diameter holes to a depth of 6 m. The pile casings sank some 5 m under their own weight, with gentle tapping required to reach their design embedment depth. Following clean-out and dewatering, a reinforcing cage, consisting of 8-24 mm diameter bars, and concrete were placed in the steel shells. The pile structural properties at time of test are summarised in Table 1.

Steel Yield Strength (shell)	f_y	=	300 MPa
Young's Modulus, Steel,	E_s	=	200 GPa
Young's Modulus, Concrete,	E_c	=	26 GPa
Concrete Compressive Strength,	f'_c	=	57 MPa
Transformed Second Moment of Area (uncracked)	I	=	0.0047 m^4
Transformed Second Moment of Area (cracked)	I	=	0.0035 m^4
Pile Cracking Moment,	M_{cr}	=	117 kNm
Pile Yield Moment (shell),	M_y	=	717 kNm
Pile Mass Per Unit Length	m	=	488 kg/m

TABLE 1: PILE STRUCTURAL PROPERTIES

Prior to installation, the steel shells were instrumented with strain-gauges at diametrically opposite locations. The gauges were protected with steel angle sections welded to the pile shells and filled with an epoxy resin. Earth pressure cells and pore pressure cells were mounted on the east pile. Rotations and deflections at ground level and deflections at the level of the applied load were recorded using displacement transducers mounted on a remote-supported reference frame. The general arrangements for the test piles and the positioning of the instrumentation is shown in Figure 2.

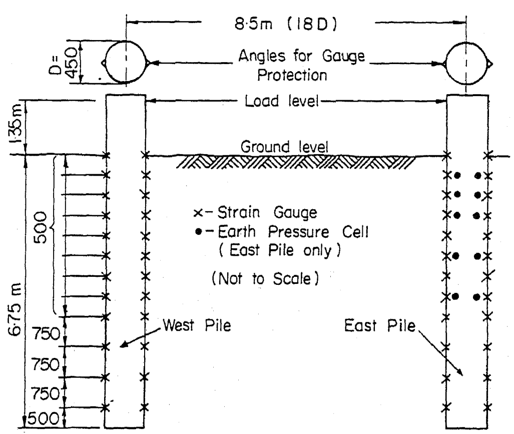

FIGURE 2. GENERAL TEST SET-UP

TEST PILE LOADING

The piles were subjected to both dynamic and slow cyclic loads. Dynamic loads were generated using a variable frequency shaking machine (machine mass 340 kg) which incorporated contra-rotating eccentric weights to give a sinusoidally varying force in the horizontal direction. Slow cyclic loads were applied by means of push/pull jacking of a strut attached to the piles at a level 1.35 m above ground level.

Initial testing involved dynamic shaking of the west pile. This was followed by slow cyclic loading of the piles at the rate of about one cycle per hour. The sequence of loading, prescribed by the nominal peak cycle load, was 10 kN, 20 kN, 40 kN (2 cycles), 80 kN (2 cycles), 120 kN (2 cycles), 160 kN (2 cycles) and 200 kN (2 cycles).

Test Data

Pile shell strain data was used to evaluate a number of profiles for the piles below ground level, as outlined in Table 2. Integration and differentiation was performed using finite difference techniques.

Diametrically opposite strains recorded at a section = e_1, e_2

section curvature, $\phi = \dfrac{e_2 - e_1}{D}$, where D is the pile diameter

moment, $M = EI\phi$

Differentiation		Integration	
shear,	$V = dM/dx$	rotation,	$\theta = \int \phi dx$
pressure,	$p = \dfrac{1}{D}\dfrac{dV}{dx}$	displacement,	$v = \int \theta \, dx$

TABLE 2: STRAIN-GAUGE DATA REDUCTION PROCEDURE

The ground level load/deflection behaviour for the east pile, as predicted from the strain-gauge data, is presented in Figure 3. Note that only a small reduction in the overall stiffness with increasing load was observed. The behaviour of the west pile was similar. Finite difference integration procedures to predict pile rotation and displacement were quite successful and the obtained deflection profiles for peak cycle loads of 40, 120 and 200 kN are presented in Figure 4. The simplifying boundary conditions of zero displacement and zero rotation at the pile tip, used in the integration procedure, resulted in some discrepancy between the predicted and recorded defections at ground level. The use of the recorded ground line deflections and rotations as boundary conditions would have eliminated this discrepancy and with hindsight would have been more appropriate boundary conditions. The corresponding moment profiles are presented in Figure 5.

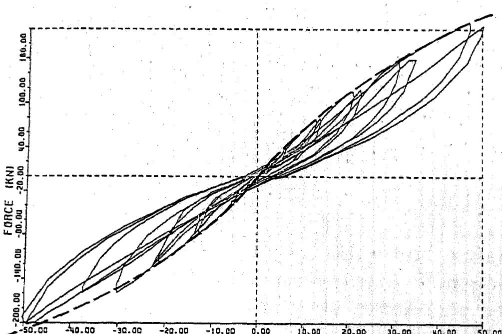

FIGURE 3. EAST PILE GROUND LINE DEFLECTION (STRAINGAUGES)

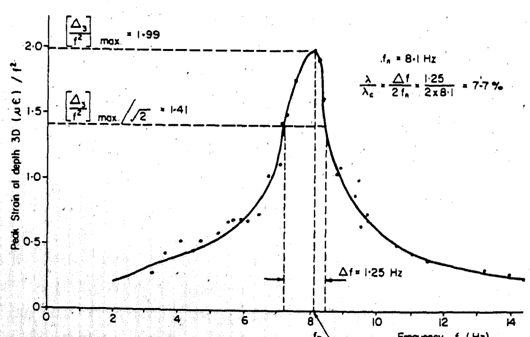

FIGURE 6 'DISPLACEMENT' RESPONSE CURVE

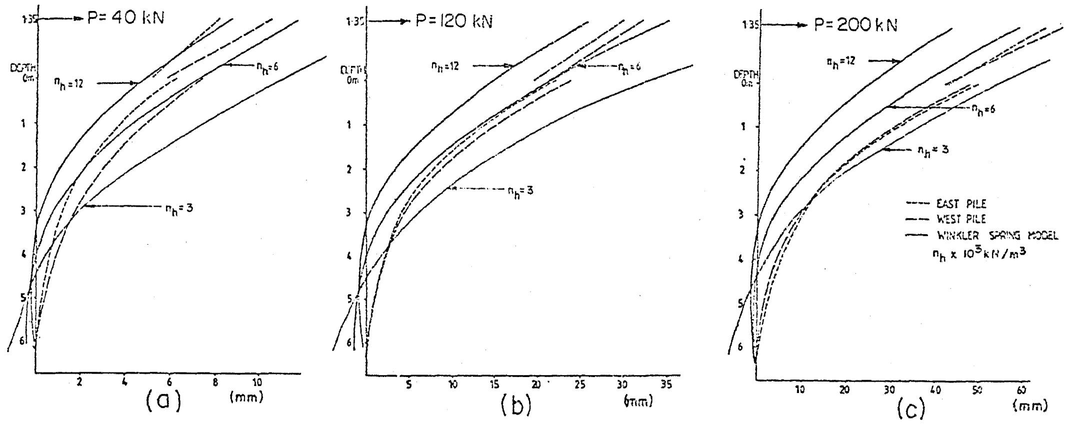

FIGURE 4. DEFLECTION PROFILES

The dynamic loading sequences were aimed at investigating the influence of load level on the dynamic response of the piles. However, only one successful suite of readings was obtained as the shaking machine was not sufficiently powerful to drive the heavier sets of eccentric weights through the frequency range of interest. The results of this successful test are presented in Figure 6. During the dynamic loading of the west pile, the soil surrounding the pile was observed to liquefy over some 100 mm around the pile. The depth of this liquefaction was not measured. It is notable that this ground disturbance had little influence on its behaviour relative to the east pile during the subsequent slow cyclic loading (see Figures 4 and 5).

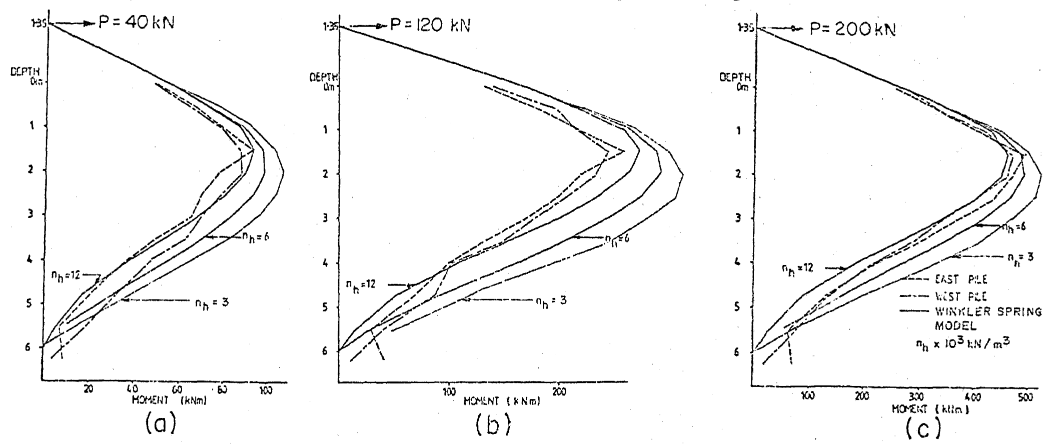

FIGURE 5. MOMENT PROFILES

EVALUATION OF SUBGRADE REACTION CONSTANT, n_h

The coefficient of horizontal subgrade reaction, k_h, is the ratio of pressure to deflection (p/y). The modulus of horizontal subgrade reaction, K_h, is the ratio of pressure to lateral diametral strain (pD/y). Terzaghi (Ref. 1) has shown that K_h is independent of pile diameter and in cohesionless soils may be be assumed to increase linearly with depth, $K_h = n_h z$, where n_h is the constant of horizontal subgrade reaction and z is the depth measured from the ground surface.

Predictions Based on Soil Investigation Data

Table 3 summarises estimates for the constant of subgrade reaction predicted from empirical relationships available in the literature. The constant listed as having been based on the average of the cyclic moduli from the Ménard pressuremeter and the initial SBP moduli must be treated with caution as it is based on very limited data.

n_h (kN/m^3)	Data	Reference
1.5×10^3	Ménard pressurement	Baquelin et al, Ref. 2 ($K_h \approx E_m$)
4.5×10^3	Avg. Ménard cyclic moduli and SBP initial moduli	Ref. 2 and Ref. 3
6×10^3	Cone penetrometer	Schmertmann, Ref. 4 ($K_h \simeq 0.8E$, where $E \simeq 1.5$ qc)
2×10^3	Loose sand (dry)	Terzaghi, Ref. 1
1×10^3	Loose sand (submerged)	Terzaghi, Ref. 1

TABLE 3: CONSTANT OF SUBGRADE REACTION FROM SOIL DATA

Predictions Based on Pile Test Data

A Winkler spring model of the test pile, using the uncracked pile section properties to represent pile stiffness, was analysed using ICES STRUDL. The soil spring constants, k_s, were evaluated using the relationship $k_s = n_h z L_s$, where L_s is the contributory length of pile. Comparison with the recorded data indicates that use of $n_h = 6 \times 10^3$ kN/m^3 predicts deflections to within 20% for all levels of loading, see Figure 4. In the Winkler spring model, no conditions of fixity (i.e. zero rotation or zero deflection) were imposed upon the pile tip.

The maximum applied jack loading of 200 kN induced a pile moment equal to approximately 70% of the pile first yield moment and produced a diametral strain (y/D) at ground level of approximately 0.1. At this level of loading, the soil pressures over the first two metres of the pile, indicated by the STRUDL model, were of the order of 6 K_p, where K_p is the Rankine passive pressure for the soil.

It was found that when the cracked section second moment of area was used to represent pile stiffness in the STRUDL model, the maximum increase in the moment and deflection profiles for the 200 kN load case was less than 5%. For the purpose of design, it would appear that the use of the uncracked section properties are appropriate, even though at this level of loading, the bending moment over a substantial proportion of the pile exceeded the cracking moment.

The subgrade reaction constant, evaluated from the test data using the expression proposed by Broms (Ref. 5 also Ref. 3) for short piles, indicated that the use of $n_h = 3 \times 10^3$ kN/m^3 gave the closest prediction for recorded ground level deflections.

Use of the equivalent cantilever method solution in the form proposed by Kocsis (Ref. 6 also Ref. 3) indicated that $n_h = 5 \times 10^3$ kN/m^3 gave the best fit with the measured pile load level deflections. Using $n_h = 6 \times 10^3$ kN/m^3 underestimated the deflections by some 12%.

Deflection estimates are generally recognised as the most uncertain aspect of soil pile interaction, with pile moments less sensitive to variation in the soil stiffness, compare Figures 4 and 5. In the above assessments for n_h, the prediction of pile deflections has been chosen as the governing criterion. The back calculated subgrade reaction constants are summarised in Table 4.

n_h (kN/m^3)	Analysis Method
6×10^3	Winkler Spring (STRUDL Model)
3×10^3	Rigid Pile (Broms, Ref. 5)
5×10^3	Equivalent Cantilever (Kocsis, Ref. 6)

TABLE 4: CONSTANT OF SUBGRADE REACTION FROM PILE TEST DATA

DYNAMIC LOAD BEHAVIOUR

The percentage of critical damping, determined from the pile response to dynamic load presented in Figure 6, is 7.7%. This representation uses the concept of "equivalent viscous damping" which has been shown to be satisfactory provided the total damping of the structure is less than about 15% of critical.

Analysis using the Winkler spring model with $n_h = 6 \times 10^3$ kN/m^3 gave a natural frequency of $f_n = 10.9$ Hz. As commented previously, liquefaction of the soil surrounding the pile developed during dynamic loading. This suggests significant softening of the soil near the ground surface. It was found that by reducing the stiffness of the Winkler springs over the top 2 m of soil by a factor of 10 (i.e. to $n_h = 0.6 \times 10^3$ kN/m^3) the natural frequency of the STRUDL model was reduced to the natural frequency (8.1 Hz) observed from the dynamic tests. This suggests that liquefaction developed to a depth of about 2.5 m for the west pile, which is consistent with the SPT profile (Fig. 1). The STRUDL model with these softened Winkler springs over the first 2 m of the pile indicated that peak deflections increased by up to 25% and peak moments by up to 15%.

CONCLUSIONS

1. The testing provided further confirmation that the use of a Winkler spring model suitably represents the behaviour of piled foundations subject to lateral loading.

2. A subgrade reaction constant, $n_h = 6 \times 10^3$ kN/m^3 was found to be reasonably representative of the saturated loose uniform silty sand at the site.

3. The use of a subgrade reaction constant independent of soil strain gave a reasonable prediction of the pile behaviour up to load levels corresponding to a pile lateral diametral strain (y/D) of 0.1 at the ground surface.

4. The use of subgrade reaction parameters based on slow cyclic loading pile tests will underestimate moments and significantly underestimate deflections of a free head pile subject to lateral loading if foundation softening occurs under dynamic loading.

5. Instrumentation used in pile tests requires careful evaluation for sensitivity and reliability. Pile casing strain-gauge location requires careful consideration.

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